

Bridge Testing – A Surprise Every Time



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Bridge Testing – A Surprise Every Time

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Abstract:	Experience with field testing of highway bridges in Ontario during recent years has shown that nearly every bridge has some aspect of behaviour which can escape the attention of even experienced bridge designers and analysts. This report lists some of the various "surprises" encountered in bridge testing which may have a significant influence on the load carrying capacities of bridges. In particular, reference is made to the behaviour of bridges with steel girders and concrete deck slabs, and of steel truss bridges. Lessons drawn from tests on these bridges are summarized, so that they can be used to advantage in the load carrying capacity evaluation. It is also shown that in some cases the appearance of a bridge can be misleading with regards to its true load carrying capacity. In such cases, field testing is shown to be the most effective means of evaluating the bridge.
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1/ Introduction

There is no better way for a bridge engineer to understand the shortcomings of the mathematical models used for design or evaluation of bridges than to investigate the behaviour of bridges through field testing. The Ministry of Transportation of Ontario (MTO) has for many years conducted a program of field testing of highway bridges which has included both static and dynamic tests. The dynamic tests were necessarily of the behaviour type, but static tests included not only behaviour tests, but also proof tests and a few ultimate load tests. More than 225 bridges have been tested in Ontario in recent years.

As a result of participation in this testing program, many lessons have been learnt and it has become obvious that design analysis customarily practiced can be in error in not one, but many different ways. Indeed, it is fair to say that virtually every bridge test had a surprise in store bringing to notice some significant aspect of bridge behaviour which had been more or less completely ignored in the evaluation analysis of the bridge.

It is now possible to draw some valuable lessons from these experiences and to explain some, although not all, of the surprises that have been encountered. These explanations, which are developed in the succeeding portions of this report, have required much deep and protracted thought. Indeed, in some cases an understanding of the behaviour was not arrived at until a long time after the test had been carried out.

Many bridges proof-tested in Ontario were able to sustain safely much larger loads than their estimated capacities as reflected by the posting loads. To illustrate this point, the example is given of the truss bridge shown in Figure 1 which had a posted vehicle weight limit of two tonnes, and yet was able to sustain safely the static weight of an approximately 70 tonne vehicle. Admittedly, a bridge posted for two tonnes should be able to sustain safely much larger statically applied loads. However, it can also be appreciated that the difference between the posted and proof loads for the bridge shown in Fig. 1 is undesirably large. It is noted that fairly large differences between the calculated and actual load carrying capacities of

bridges have been encountered in so many instances that they are no longer included in the "surprises" of bridge testing.

The large differences between the actual and calculated capacities of a bridge are caused by errors in the mathematical modelling of the structure for the purpose of both force and strength analyses. Some of these errors are brought to light by the various surprises found in bridge testing. Fortunately, in most cases, the errors in the modelling of the structure, are on the safe side. However, as is also shown in this report, there are a few isolated cases in which the error in modelling may be on the unsafe side.

The purpose of this report is to record some of the significant surprises encountered in bridge testing, so that the lessons drawn from them may be used with advantage in the evaluation of the load carrying capacities of existing bridges.

In order to give at least some semblance of order, the catalogue of the diverse surprises is classified in this report with respect to specific types of bridges, and in one instance, under the heading "misleading appearances".

2/ Slab-on-Girder Bridges

2.1/ Enhanced Flexural Stiffness

Tests on slab-on-girder bridges have shown consistently that these structures are usually much stiffer flexurally than what is predicted by calculations. The unaccounted-for flexural stiffness of these bridges manifests itself in dynamic tests in the form of measured frequencies that are larger than the calculated ones (e.g. Biggs and Suer, 1956; Edgerton and Beecroft, 1956; Wright and Green, 1962; and Cantieni, 1983). In static tests the measured deflections and girder strains are found to be much smaller than calculated, again indicating the bridge to be stiffer in flexure than assumed.

The reason for the discrepancy between calculated and actual stiffnesses of slab-on-girder bridges has often been attributed to (a) a very high modulus of elasticity of concrete, (b) interaction of the girders with secondary components such as barrier

walls and wind bracing, and (c) the presence of a high degree of composite action between the girders and deck slabs even when they do not have any mechanical shear connection between them. However, unintentional composite action in bridges of non-composite type of design cannot be regarded as a major reason for the discrepancy mentioned above. This is because even those bridges which do have mechanical shear connections between the deck slab and girders have been found to be stiffer than shown by calculation, notwithstanding that the stiffness calculated were based upon fully composite action. An example of such a bridge is the North Muskoka River bridge which is one of 29 bridges tested dynamically by Billing (1982). The observed frequencies of this bridge showed that it had a higher flexural stiffness than could be rationalized even by a high modulus of elasticity of concrete and the inclusion of barrier walls and wind bracing in the calculations.

The North Muskoka River bridge was retested by Bakht (1988a) to find the reason for the discrepancy between the actual and observed stiffnesses of the girders. This relatively new bridge has five apparently simply-supported girders resting on bearings of the elastomeric type. The retest of the bridge showed that even in the presence of elastomeric type of bearings, the girders develop enough bearing restraint force to reduce the applied load moments at midspan by a significant margin. The bearing restraint forces for two particular load cases, calculated from girder strains at sections close to the bearings are shown in Fig. 2. It can be appreciated that these forces are larger than could be expected to develop in elastomeric bearings. It is noted that in the Figure the discrete points representing bearing restraint forces in girders are joined by continuous curves; this is done merely to enhance the readability of the figure. This technique has also been used in other figures.

Several other tests on slab-on-girder bridges (e.g. Bakht, 1988b, and 1988c) have confirmed the presence of large bearing restraint forces which tend to stiffen up these bridges. It is appropriate to mention that attention to bearing restraint forces was drawn by Biggs and Suer (1958). They calculated the effective flexural rigidity of several slab-on-girder bridges by using both measured static load deflections at midspan and also measured frequencies. When it was found that the two procedures gave different results, Biggs and Suer (1958) conceded that this discrepancy could not be explained at that time, but offered as a possible explanation the

interaction of secondary and dynamic components, and the presence of a frictional force developed between the girders and their bearings.

2.2/ Composite Action In Non-Composite Bridges

Measurements of strains over the depth of steel girders have shown that some degree of composite action exists between the deck slab and girders even in the absence of mechanical shear connectors. Some researchers have even suggested that this unintentional composite action be utilized in the load carrying capacity evaluation of the bridge. An ultimate load test of a slab-on-girder bridge (Bakht, 1988c) has, however, shown that in the absence of mechanical shear connectors, the composite action between a girder and the deck slab breaks down completely as the failure limit state for the girder is approached. This point is illustrated in Fig. 3 which shows the load level plotted against the position of the neutral axis in two of the most heavily loaded girders of the bridge mentioned above. It can be seen that at low levels of the load, the neutral axis in both girders is well above the neutral axis of the steel girder alone, indicating the presence of some composite action. However, as the load approaches the failure load of the girder, the neutral axis drops to the level of the neutral axis of the girder.

2.3/ Failure Mode of Deck Slab

From the ultimate load test mentioned above, it was found that the behaviour of the deck slab at the ultimate limit state of the bridge was totally different from that for which deck slabs are usually designed. As shown in Fig. 4, the slab developed a herring bone type of crack pattern at the failure of the bridge. Bakht (1988c) explains with the help of Fig. 5 that these cracks develop due to the presence of composite action between the deck slab and some of the girders even if it breaks down completely for some other girders.

It should be noted that in the ultimate load test mentioned above, a uniformly distributed load covering only a partial portion of the bridge was applied. Perhaps because of this, the deck slab remained serviceable even after the development of diagonal cracks.

2.4/ Transverse Load Distribution in Non-Composite Bridges

In bridges with no mechanical shear connection between the deck slab and girders, a deterministic transverse load distribution analysis is made difficult, if not impossible, by the uncertain degree of participation of the deck slab with the various girders. This point is illustrated by the example given in Fig. 6 which is drawn from a proof test reported by Bakht (1988b). In this figure, actual values of distribution factor (DF) for midspan girder moments are compared with those obtained from two different analyses by the semicontinuum method (Jaeger and Bakht, 1988). The girders were assumed to be fully composite with the deck slab in one analysis, and non-composite in the other. It can be seen in Fig. 6 that, contrary to engineering judgement, the two sets of analytical results do not bracket the actual behaviour. This phenomenon emerges from the fact that the uncertain degree of composite action results in uncertain values of the effective flexural rigidities of the girders.

It can be concluded readily from the DF values for midspan moments plotted in Fig. 6 that girders 2 to 4, counted from the left-hand girder, are much stiffer than girders 5 and 6. Unless actual values of girder stiffnesses are known, no deterministic method of analysis can realistically predict the transverse load distribution pattern of these bridges.

2.5/ Summary of Surprises, Slab-on-Girder Bridges

The relevant lessons learnt from the testing of slab-on-girder bridges are summarized in the following:

- a/ Bearing restraint, even in bridges with elastomeric bearings, makes slab-on-girder bridges stiffer than they are calculated to be by calculations in which this restraint is neglected.
- b/ In bridges without any mechanical shear connection between the deck slab and girders, any composite action which may exist at service load levels breaks down completely at failure.
- c/ The transverse load distribution in non-composite bridges is so random that it cannot be accounted for realistically by deterministic analyses alone.

3/ Steel Truss Bridges

Simply-supported steel truss bridges are usually simple to analyse because of the limited number of paths that a load can take. Tests on these bridges have shown, however, that even these bridges have certain aspects of behaviour which may surprise a bridge designer or analyst.

3.1/ Load Sharing between Components

It has been noticed in several tests that in rigid-jointed steel trusses, two component members of a tension chord share the load unequally, with the component towards the inside of the bridge usually carrying a higher share of the load. Since the difference between the loads shared by the two components is usually not large, and since the steels used in most of the rigid-jointed trusses are ductile, it is assumed, justifiably, that at failure the two components would eventually share the load equally.

When relatively old bridges with truly pin-connected steel trusses were tested recently, it was found that the two component members of the various tension chords of the trusses shared the load very unevenly. This phenomenon is illustrated in Figs. 7 (a) and (b) which are reproduced from Bakht and Jaeger (1987). It can be seen from these figures that in some cases, the outer component took almost all the load and in others the inner component sustained all the load. Bakht and Jaeger (1987) have concluded that because of the brittle nature of the steels used in these relatively old pin-connected trusses, it may be prudent to include only one component member in the calculation of the capacity of a tension chord.

3.2/ Interaction of the Floor System with the Bottom Chord

The bottom chord strains plotted in Fig. 7 (a) were found to be smaller, by a factor of about 15, than the strains which would have occurred if the chord sustained all the force itself. It is obvious that in this case, the stringers of the floor system sustained a large share of the tensile force in the two trusses. The observation that, in pony-truss and through-truss bridges, the floor system takes a large portion of

the tensile force in the two trusses has been made so many times that it has virtually become a cliché. Nevertheless, a surprising feature was observed in a test whose results are reported in Fig. 7 (b).

The surprising feature relates to the bottom chord strains in the panel closest to the right hand support of the truss. It can be seen in Fig. 7 (b) that the strains in this panel are about 15 times larger than the strains in the adjacent panel. Since the two panels have components of the same section, it is obvious from simple statics that the bottom chord force in the two panels should be very nearly the same. The fact that the total strains in the bottom chords of the two panels are so significantly different from each other suggests that the floor system does not participate with the bottom chord in the end panel.

This behaviour is explained as follows by Bakht and Jaeger (1987). All stringers of the floor system of the bridge shown in Fig. 7 (b) are connected to the truss nodes in such a way that the bottom chord between adjacent nodes cannot deform without engaging the stringers. As shown in this figure, the bridge has a small approach span which is formed by extending the stringers of the truss floor system beyond the pier. Because of this extension, the floor system does not have a floor beam at the end node, as it does for all other nodes. Accordingly, the bottom chord in the end panel, by deforming independently of the stringers, is called upon to sustain all the tensile force of the truss.

3.3/ Local Failure of Compression Chord

As described by Bakht and Csagoly (1979), a steel through-truss bridge, called the North West Arm bridge, was tested to failure; the primary purpose of the test was to validate a method of predicting the in-plane buckling behaviour of trusses. To ensure that the bridge did not fail in tension, some of the tension chord members were reinforced. Under the test load the bridge failed prematurely due to the local buckling of the cover plate of the built-up section of an inclined chord. This local failure, which is shown in Fig. 8, resulted in the failure load of the bridge being about one-third less than the predicted failure load.

It was discovered that this unexpected source of weakness in the strength of the compression chord was the building up of rust between the sparsely riveted cover plate and the channel sections. This building up of rust caused the cover plate to ripple between the rivets, and consequently to be subjected to markedly eccentric compressive forces, thereby reducing its capacity to sustain compressive loads.

3.4/ Floor System Sharing of the Applied Load

It is very useful to perform simple checks in which the total applied moments in a simply-supported right bridge are compared with those computed from measured responses such as strains. In a test on a relatively short steel pony-truss bridge with a simply supported right span, it was found that the moments taken by the trusses constituted only about 92 percent of the applied load moments. A careful search for the reason for this discrepancy revealed that the floor system, which has continuously connected stringers throughout the length of the bridge, has a flexural stiffness equal to about one-eighth of the combined stiffness of the two trusses. The testing on this particular bridge is described by Tharmabala (1985). It is obvious that the floor system of the bridge under consideration is able to take about eight percent of the applied load directly by spanning between the truss supports, thus providing some relief to the trusses.

3.5/ Component Interaction

The beneficial effect of component interaction in truss bridges is often disregarded as being insignificant. Confirmation that the effect of interaction is not always small was provided by a recent test on a 100 m span deck truss type of arch bridge. The bridge has a large number of transverse floor beams of the same cross-section, the strengths of which were investigated by a recent test. The bottom flanges of these beams were instrumented with strain gauges at their respective midspans.

As shown in Fig. 9, it was found that under similar loading, the four beams close to one of the truss supports showed consistently higher strains than the other two instrumented floor beams. The reason for this unexpected behaviour became obvious after an inspection of the cross-frames under the various floor beams. The configurations of the cross-frames are shown in Fig. 10. It can be seen that the

four floor beams close to a truss support span directly between the trusses as was assumed in the evaluation and probably would have also been assumed in the design calculations. However, the other floor beams cannot span directly between the trusses due to the double X arrangement of members in the cross frames. It is because of the integral nature of these floor beams with the cross-frames, that the measured strains were considerably smaller than they would otherwise have been. Further details of the test under consideration are given by Tharmabala (1985).

3.6/ Summary of Surprises, Steel Truss Bridges

The various surprises encountered in the testing of steel bridges, and the lessons learnt therefrom, are listed in the following:

- a/ The various components of the tension chords of pin-connected trusses share loads so unevenly that only one component should be considered in the calculations for bridge evaluation.
- b/ In pony-truss and through-truss bridges, the floor system participates with the bottom chords of the trusses only if the stringers are connected effectively with all the nodes of the trusses.
- c/ In calculating the capacity of a compression chord, account should be taken of the potential source of weakness introduced by the rippling of the cover plate between rivets, which may be caused by the building up of rust between the cover plate and the other components of the compression chord.
- d/ In some bridges advantage can be taken of the fact that floor systems of substantial flexural rigidity can themselves take a sizable portion of the load by spanning directly between the truss supports.
- e/ Component interaction can in certain cases be used to advantage in upgrading the analytical bridge capacity.

4/ Misleading Appearances

Apparent evidence of deterioration can sometimes be misleading. Many apparently severely deteriorated concrete bridges have been found to be capable of carrying normal loads, and their load carrying capacities cannot, to date, be explained analytically even after assessing the concrete strengths by tests on samples removed from the bridge. Notwithstanding such cases, two examples are presented in the following for which the behaviour can be explained readily.

The first example is that of the cantilever sidewalk of an old arch bridge, which seemingly is supported by cast iron brackets as shown in Fig. 11. Many of these brackets were so cracked that their capacity to sustain the sidewalk loading was suspect. A load test on the sidewalk with loads about five times the design load showed that the brackets did not participate at all in carrying the sidewalk load. The brackets were, in fact, entirely ornamental and the cantilevered deck slab is more than capable of sustaining the sidewalk loading.

The second example is that of an old, 5.5 m span slab-on-girder bridge with steel girders and concrete deck slab with monolithic parapet walls. The cross-section of the bridge is shown in Fig. 12 together with the relevant dimensions. Recently, it was found that an outer girder somehow moved away from under the bridge. This led to an analytical evaluation of the load carrying capacity of the bridge which concluded that the girders were not capable of even carrying the dead load of the bridge. The bridge drawings were not available. However, since the bridge was known to have carried normal traffic for a number of years, it was not closed but was reluctantly restricted to vehicles having gross weights of less than two tonnes.

A very brief test on the bridge with a 22 tonne vehicle showed that only a very small portion of the applied loading was being taken by the girders. As shown in Fig. 12, the bottom flange stresses in the girders were only a fraction of the stresses which would have been induced if the girders had sustained the applied loading all by themselves. It was concluded that the bridge was not a slab-on-girder bridge at all; it was, in fact, a concrete slab bridge with upstand beams. The "girders" were only part of the framework, and were left behind after the construction.

The above two examples underline the fact that in some cases appearances can be misleading, and that for such cases the most effective means of establishing the true behaviour of the bridge is by field testing.

Conclusions

This report lists a number of significant "surprises" encountered during the testing of highway bridges in Ontario. These surprises confirm that some aspects of bridge behaviour, because of never entering into design considerations, can escape the attention of even experienced bridge designers and analysts. Some of the surprises may have a significant bearing on the load carrying capacity of a bridge, while others may have a relatively minor effect. From the examples given in this report, it is clear that in most cases, the load carrying capacities of bridges are higher than obtained by usual calculations. However, there are some cases in which the load carrying capacity of a bridge can be lower than expected. It is, therefore, necessary that a catalogue be prepared of those aspects of bridge behaviour which do not usually enter into design considerations and which may have a significant influence on the ability of the bridge to carry loads. This report can be regarded as a contribution towards the development of such a catalogue.

With the help of two specific examples, it is shown that in some cases the bridge behaviour may be so deceptive that it can be understood only through a field test. For such cases, no amount of refinement in the analysis can yield the true load carrying capacity of the bridge.

References

- Bakht, B. (1981). "Testing of the Manitou Bridge to Determine Its Load Carrying Capacity." *Can. J. Civ. Eng.* 8(2), 218-229.
- Bakht, B. (1988a). "Observed Behaviour of a New Medium Span Slab-on-Girder Bridge." Structural Research Report, SRR-88-02, Ministry of Transportation, Downsview, Ontario, Canada.
- Bakht, B. (1988b). "Testing of an Old Short Span Slab-on-Girder Bridge." Structural Research Report, SRR-88-01, Ministry of Transportation, Downsview, Ontario, Canada.
- Bakht, B. (1988c). "Ultimate Load Test on a Slab-on-Girder Bridge." Structural Research Report, SRR-88-03, Ministry of Transportation, Downsview, Ontario, Canada.
- Bakht, B. and Csagoly, P.F. (1979). "Bridge Testing." Structural Research Report, SRR-79-10, Ministry of Transportation, Downsview, Ontario, Canada.
- Bakht, B. and Jaeger, L.G. (1987). "Behaviour and Evaluation of Pin-Connected Steel Truss Bridges." *Can. J. Civ. Eng.* 14 (3), 327-335.
- Biggs, J.M. and Suer, H.S. (1956). "Vibration Measurements on Simple-Span Bridges." Highway Research Board Bulletin 124, Highway Research Board, Washington, D.C..
- Billing, J.R. (1982). "Dynamic Loading and Testing of Bridges in Ontario, 1980." Proceedings, International Conference on Short and Medium Span Bridges, Vol. I, Toronto, Canada, pp 125-139.
- Cantieni, R. (1983). "Dynamic Load Tests on Highway Bridges in Switzerland, 60 Years Experience of EMPA." EMPA, Swiss Federal Laboratories for Materials and Testing Research, Report No. 271, Dubendorf, Switzerland.

Tharmabala, T. (1986). "Performance Studies of Keewatin Channel Bridge."
Structural Research Report SRR-85-05, Ministry of Transportation,
Downsview, Ontario, Canada.

Wright, D.T. and Green, R. (1962). "Highway Bridge Vibrations, Part II, Ontario
Test Program," Department of Civil Engineering, Queen's University,
Report submitted to the Ontario Ministry of Transportation.



Figure 1/ Bridge posted for 2 t, and carrying a static load of about 70 t

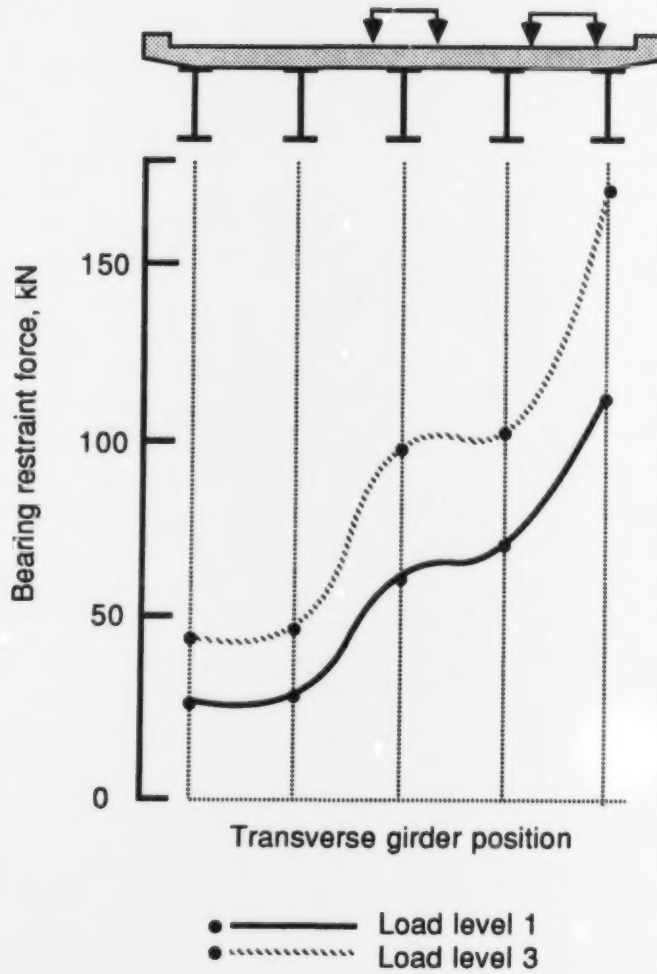


Figure 2/ Bearing restraint forces in a slab-on-girder bridge with elastomeric bearings

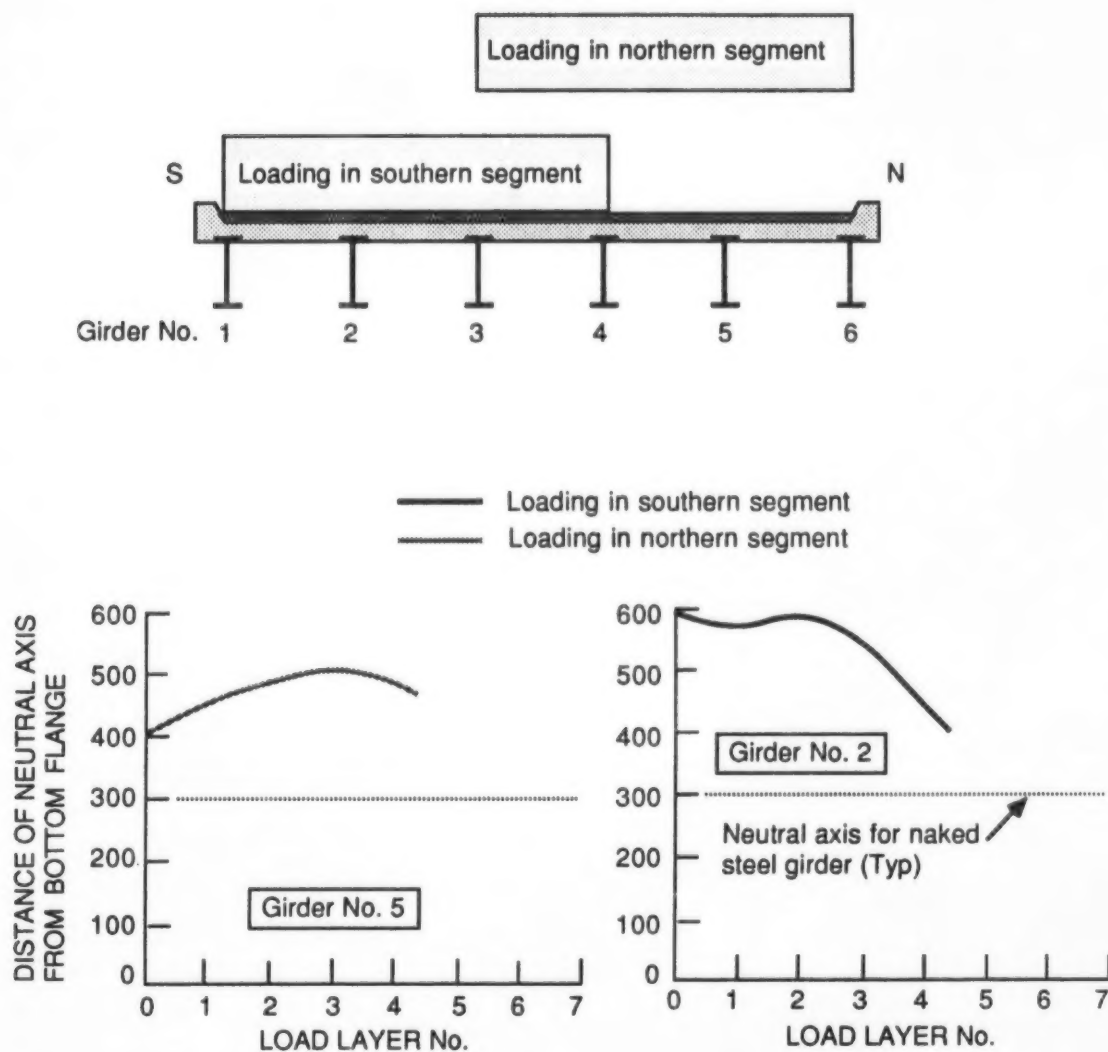
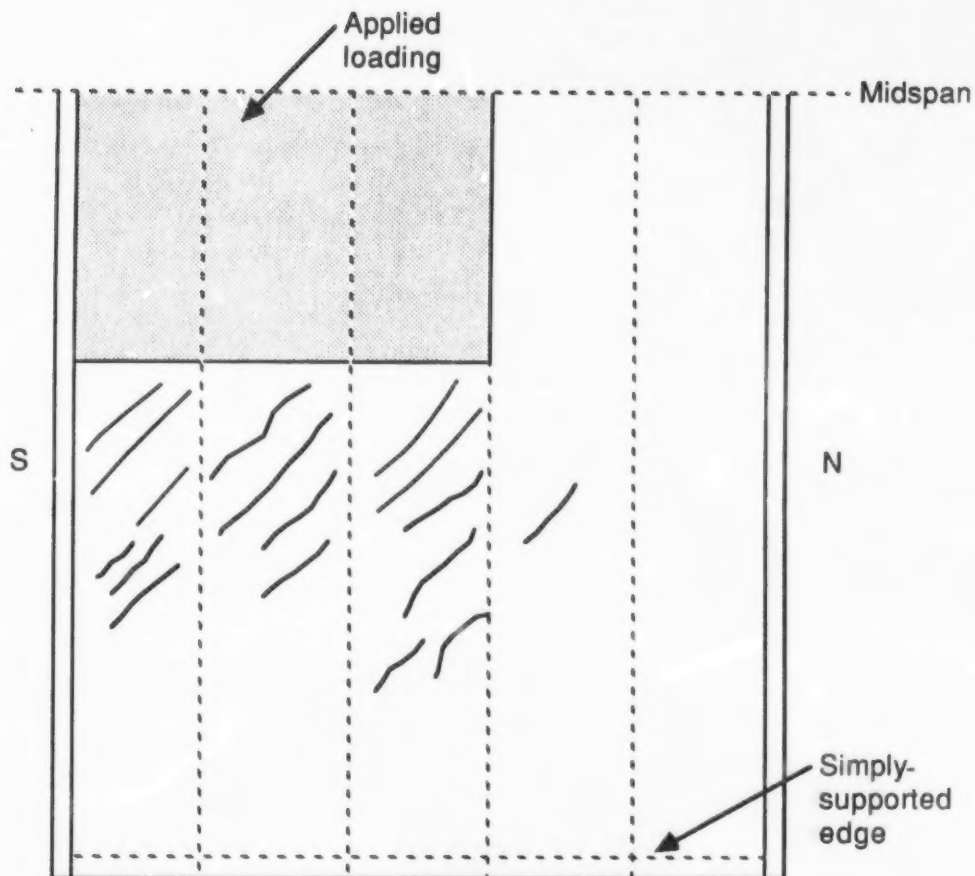


Figure 3/ Effect of load level on the position of the neutral axis in a girder of a non-composite bridge



NOTES:

1. Crack pattern was symmetrical about the midspan line
2. Cracks on the underside of the slab, shown as viewed from the top

Figure 4/ Crack pattern in the deck slab of a slab-on-girder bridge at the failure of the bridge

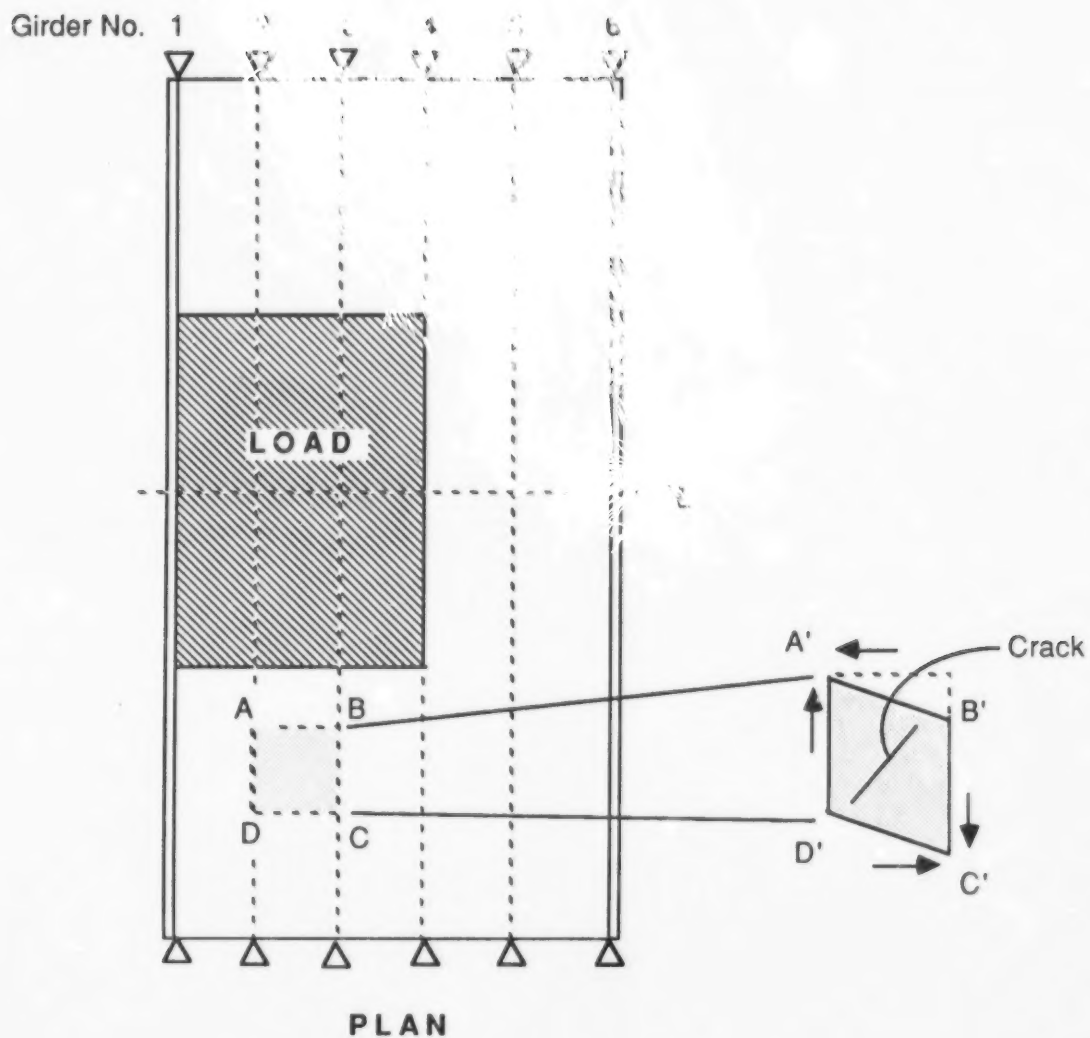


Figure 5/ Mechanics of the development of diagonal cracks in the deck slab of a slab-on-girder bridge

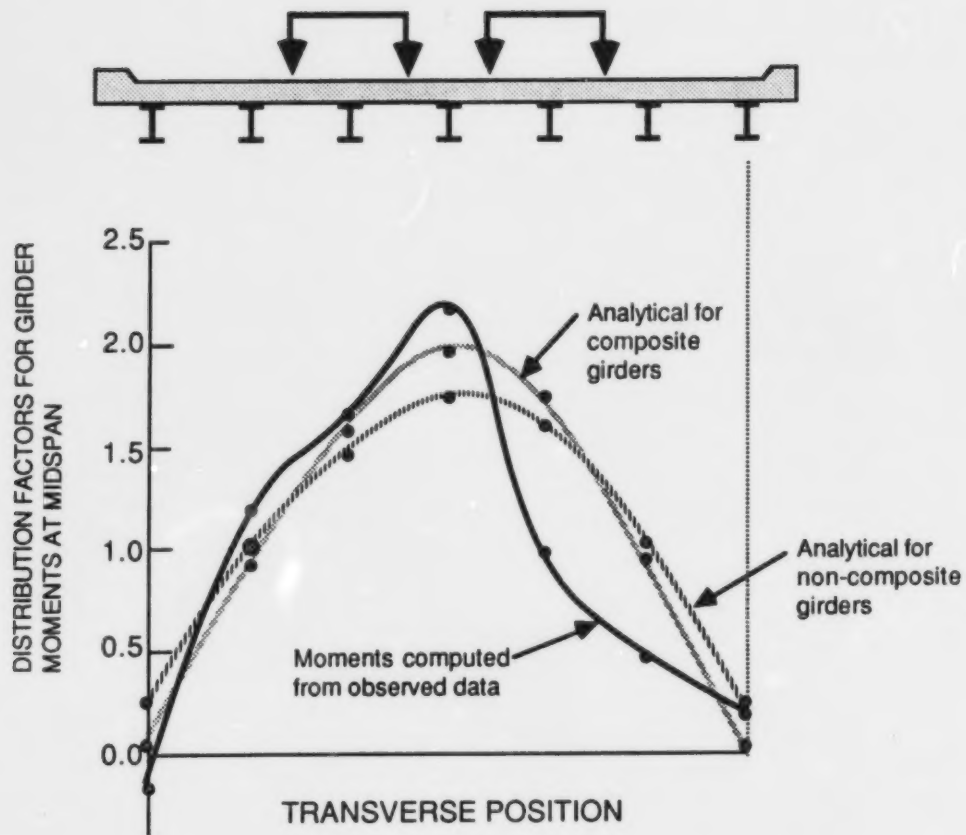


Figure 6/ Actual and analytical distribution factors for midspan girder moments

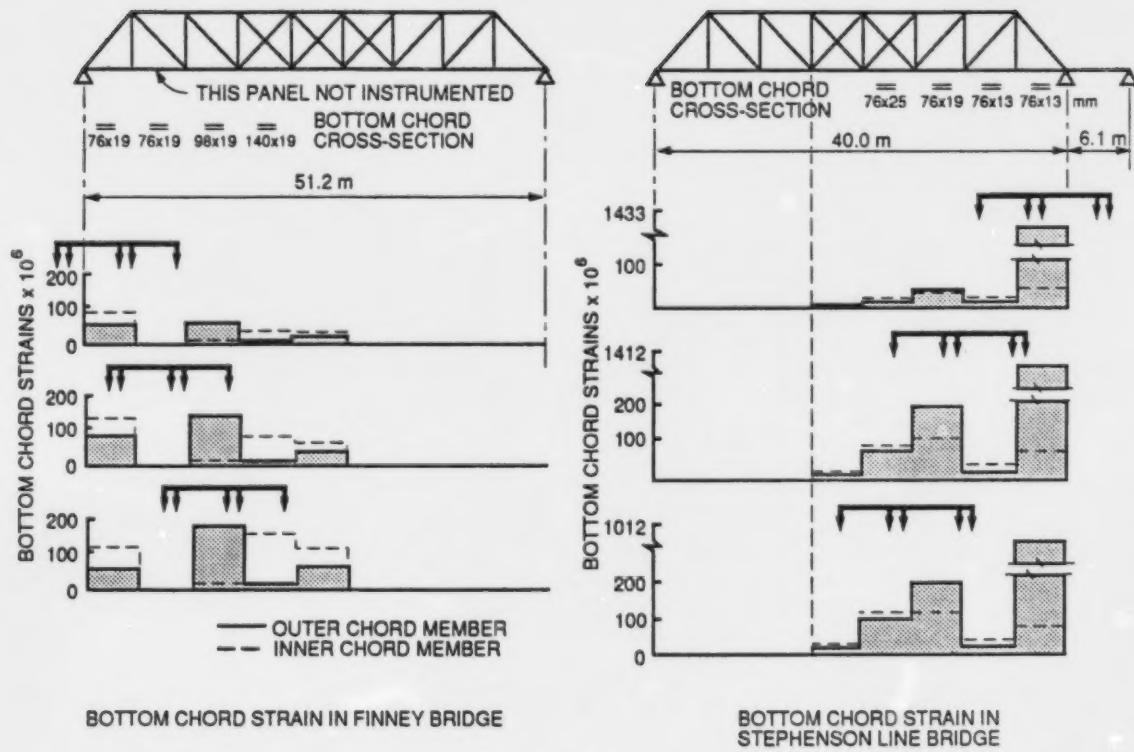


Figure 7/ Strains along the bottom chord of the trusses of two different bridges

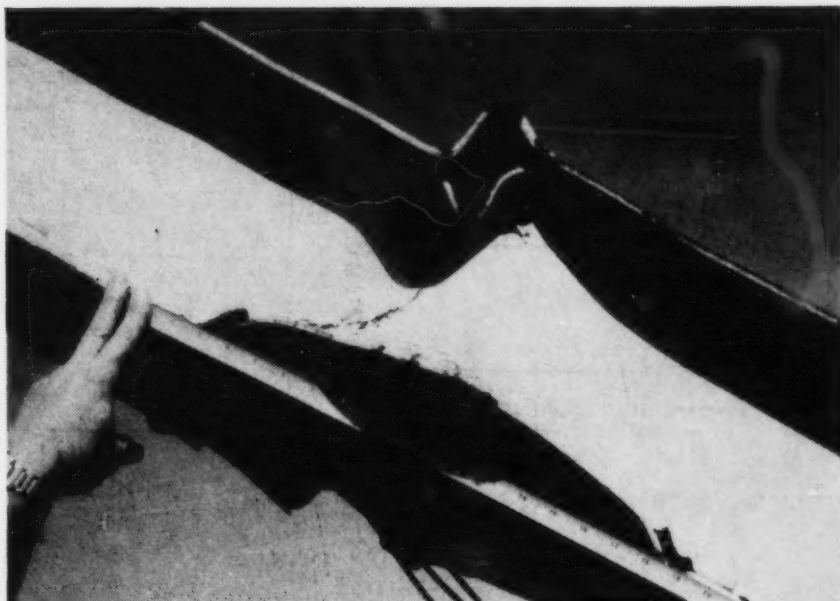


Figure 8/ Local failure of a compression chord of a steel truss

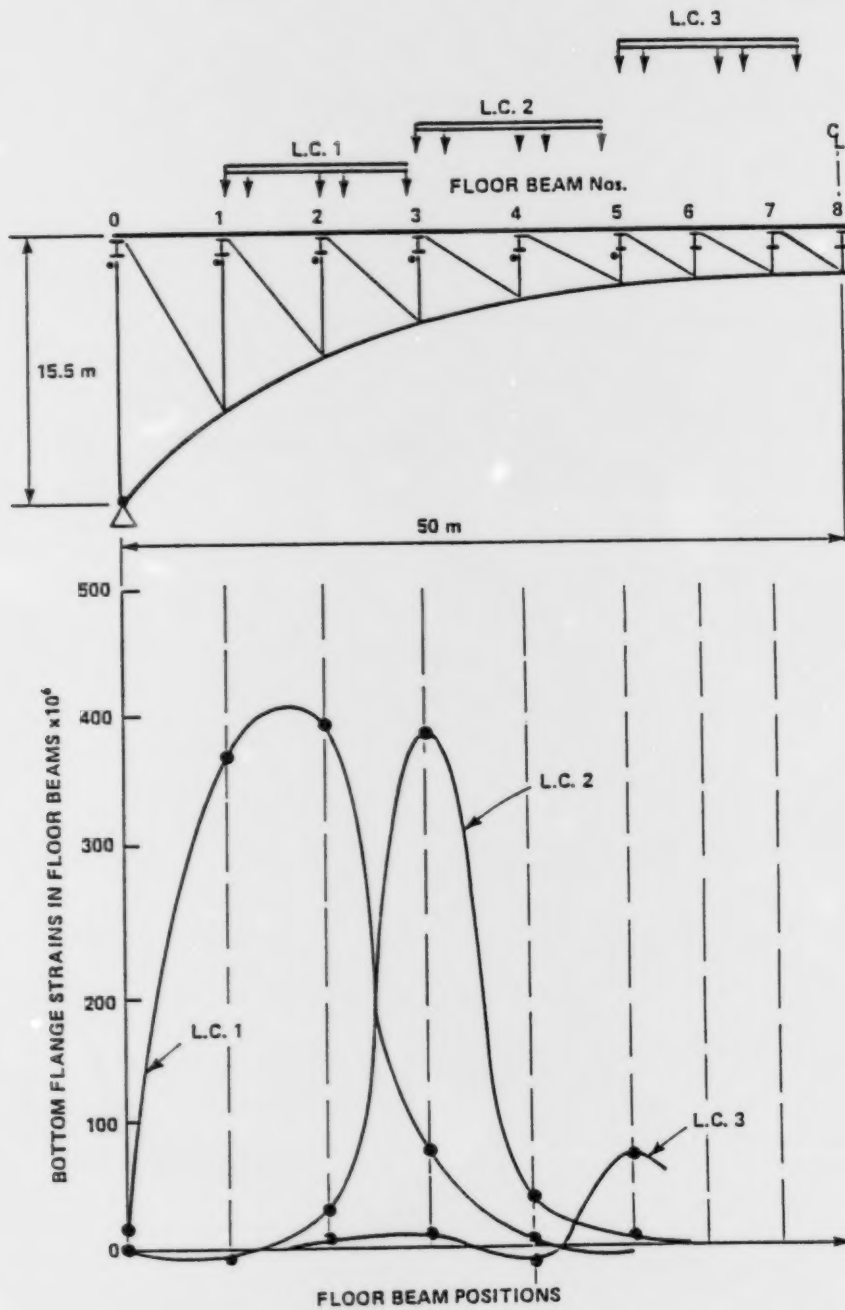


Figure 9/ Strains in the bottom flanges of floor beams at their midspans

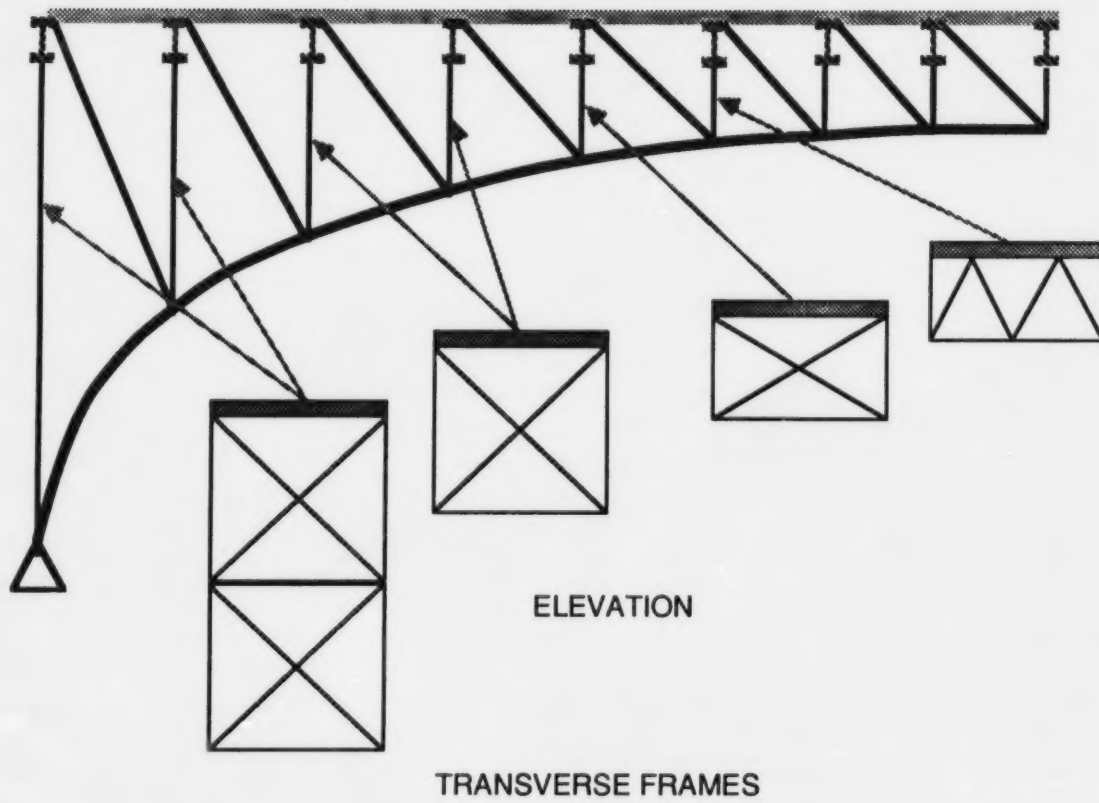


Figure 10/ Details of cross-frames



Figure 11/ Cast Iron brackets apparently supporting part of the sidewalk

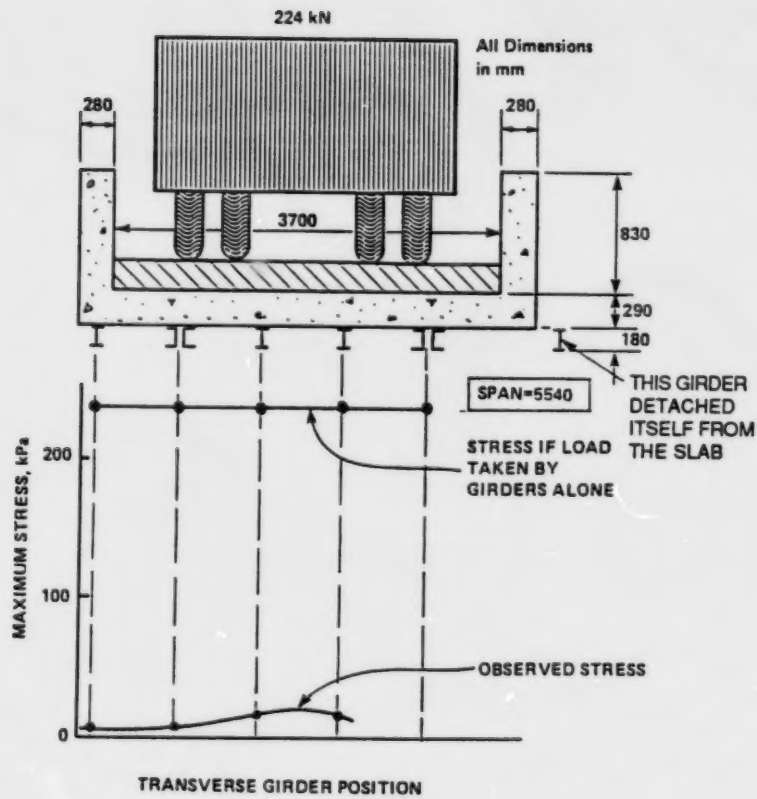


Figure 12/ Bottom flange strains in girders of a slab-on-girder bridge